DYNAMIC ELASTIC ANALYSES IN THE DESIGN OF TYPICAL NEW ZEALAND HIGH-RISE BUILDINGS.

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SYNOPSIS

In this paper the structural features of ten recently designed multistory buildings are described. The methods used to predict the elastic dynamic characteristics are summarised and the results of the theoretical analyses are presented. The experimental techniques used to determine the dynamic properties of those of the buildings which have been completed are briefly outlined and the results of the tests are listed.

The reasonable correlation which exists between the predicted and measured properties allows an assessment to be made of the validity of the assumptions made at the design stage.

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INTRODUCTION

The dynamic nature of the seismic design problem is recognised in the New Zealand Building code.(1) Equivalent static design forces may be determined using the first mode period together with a specified design spectra. Alternatively a more precise form of dynamic analysis may be required for special structures and can be accepted for any structure. In this more comprehensive approach account is not normally taken of more than the first three modes of vibration; an elastic response spectrum(2) based on averaged and smoothed single mass resonator responses to the acceleration recorded in eight major strong motion earthquakes is used and the maximum responses are assumed to be the square root of the sum of the squares of the modal responses.

The Code requires that the first mode period shall be determined from properly substantiated considerations, the simplest of these requiring the establishment of the top story deflection under a prescribed lateral loading pattern. Hence irrespective of which of the two allowable design approaches is used the determination of the lateral stiffness of the structure becomes unavoidable.

Considerable difficulty is experienced in justifying the idealisations which are essential if stiffness analyses of practical buildings are to be undertaken at all. Consequently doubts may arise regarding the validity of the computed properties.

Experimental investigations of the behaviour of full-scale structures enable critical examination to be made of the design assumptions. In particular simple non-destructive tests can readily reveal certain important properties of buildings which may be used to assess the value of the predictions made at the design stage.

THE BUILDINGS EXAMINED

(i) The Auckland Customs House.

The new customs house, to be built on reclaimed land on the Auckland waterfront, will consist of a basement and twelve main floors, each of approximately 8,000 square feet gross area. The centre core section will project four floors higher, as shown in figure 1, and the overall height of the building will exceed 200 feet. The structure is to be of reinforced concrete throughout except for the top story and roof section which will be steel framed.

The main structural core is to be supported on ten 72 inch diameter piles, and the columns of the perimeter frame will each be carried on 54 inch diameter piles. All the piles are to be founded on Waitemata Sandstone 54 feet below basement level.

(ii) Fray Flats

The Fray Flats will consist of an almost symmetrical three by six bay rectangular rigid reinforced concrete frame having thirteen floors, each of 4,000 square feet area, above ground level.

The total height of the building will be 127 feet and the framework is to be supported by a very stiff pile and shear wall substructure founded directly on bed rock.

(iii) Jellicoe Towers

The Jellicoe Towers block of eighteen flats is a reinforced concrete eighteen story building, each floor being of 1400 square feet area. Structurally it consists of a service core, rectangular in plan and situated on the south side of the building as shown in figure 2, with 6 inch concrete slab floors supported by the core and the perimeter load bearing walls.

The building is founded directly on weathered greywacke rock. The raft foundation extends over the full area of the building and is connected to the main vertical structural elements through the raft beams.

(iv) *Jerningham Apartments*

The Jerningham Apartments building has sixteen floors with reinforced concrete spandrel beam frames around the perimeter. These frames provide all the effective lateral structural strength, only light internal columns being incorporated between the simple slab floors.

A machine room is situated at roof level and the southern section of the building has one less story than the northern part. The sloping site necessitated a departure from the typical framing layout at the bottom of the structure as shown in figure 3(a). A typical floor, shown in figure 3(b), has a total area of 5,000 square feet divided into four apartments.

(v) Reserve Bank.

The structural steel framework of the Reserve Bank is symmetrical in plan, having five bays in the longitudinal direction and three in the transverse, all of 325 inch centre line to centre line width as shown in figure 4. There are fourteen main floors each of approximately 11,000 square feet, and there is a set back section at the top of the building.

The basement and sub-basement structure consists of massive reinforced concrete vaults and is founded on firm variable weathered greywacke alluvial soils.

(vi) Terrace Chambers

The Terrace Chambers office building is of reinforced concrete construction having fourteen stories, each of 3,750 square feet and of 120 inch interstory height. Structurally it consists of two tower elements on the northern and southern faces on the eastern and western ends as shown in figure 5. Apart from the spandrels the 7 inch thick floor slabs provide the only structural connection between the towers and frames other than that furnished by the main ground beams which interconnect the elements at foundation level and which form part of the foundation raft. This raft extends over the entire area of the building and is founded directly on weathered greywacke rock.

(vii) Canterbury University Zoology Building

The Zoology building is a reinforced concrete composite frame and shear wall structure. Each floor is of 8,000 square feet area and there are six stories in the 85 feet high block. The structural layout is shown in figure 6. The continuous strip footings under the outer walls and the transverse shear walls are founded on well rounded gravel which extends continuously for at least 70 feet below ground level.

(viii) Canterbury University Chemistry Building

The Chemistry building is similar in structural style to the Zoology one but is considerably larger. It is 246 feet long and 49 feet wide and has eight floors with mechanical plant rooms at the roof level 100 feet above ground level.

The foundation details and site conditions are similar to those of the Zoology block.

(ix) Canterbury University Physics Building

The Physics Building is of similar structural form to the Zoology and Chemistry blocks and has similar foundation details and site characteristics. It is 193 feet long but of the same height and width as the Chemistry building.

(x) Wellington Girls' College

This reinforced concrete framed structure will have six stories each of approximately 4,400 square feet area. It will have eleven bays in the longitudinal direction and one in the transverse direction. The frames will be founded on a continuous strip footing on a highly preconsolidated clay.

ANALYSIS PROCEDURE

The elastic stiffness properties of the elements of the idealised structures were combined to form building stiffness matrices, the terms of which relate joint displacements and rotations to unit lateral loads applied at the joints.

The dynamic analyses involved iterative solutions of the equations obtained from combinations of the elastic and harmonic motion conditions.

This general method was applied to all the buildings analysed but the unusual characteristics of some of the structural configurations necessitated attention being given to particular considerations as outlined below.

(a) The Auckland Customs House.

In the case of the Auckland Customs house the effect of the pile foundation on the building's dynamic properties was of particular interest at the design stage. It was evident that a major contribution to the lateral flexibility would be made by the displacement of the piles.

Preliminary investigations of the relative lateral stiffnesses of the central tower and the frame system indicated that the tower will resist almost all the applied loads.

The properties of the central tower, assuming it to be fully fixed at its base, were first calculated. Next the behaviour of a completely rigid tower on the proposed pile foundation was examined. The element flexibilities were then added in order to determine the properties of the pile supported tower.

The fragmentation of the analysis procedure in this manner facilitated investigation of the consequences of varying certain properties, in particular the effective length of the piles, on the overall characteristics of the building(3).

(b) Jellicoe Towers.

Since the centres of mass and rigidity do not coincide in the Jellicoe Towers structure, the coupling between the translational and torsional motions had to be considered at the design stage.

No particular difficulty is involved in extending the stiffness matrix, matrix iteration approach to include torsional components of motion in the normal mode analysis procedure(4) but extra degrees of freedom must necessarily be taken into account.

The Jellicoe Towers building is almost symmetrical about a north-south axis whereas the eccentric position of the main structural element does require consideration of torsional effects when an east-west movement is envisaged(5).

(c) Jerningham Apartments.

The only complicating factor in establishing the dynamic properties of the Jerningham Apartments arose from the offset in each of the east and west face frames (figure 3(b)).

Consideration of each part of the frame acting independently and for the two sections acting together led to confident anticipation that the two portions of the frames on the eastern and western faces will exhibit satisfactory composite action(6).

The differences between the frames on the northern and southern faces of the building were considered sufficient to warrant an investigation of the importance of torsional movements. The analysis allowing for torsion showed that only insignificant changes in the predicted first mode dynamic properties were introduced.

The configuration of the Jerningham Apartments building necessitated account being taken of the flange action of the exterior frames which are transverse to the loading direction. This was conveniently done when setting up the frame stiffness matrices.

(d) Terrace Chambers.

The Terrace Chambers building was considered to be sufficiently symmetrical to justify the neglect of torsional considerations.

However the composite nature of the structure necessitated the adoption of the following procedure when computing the lateral flexibility properties of the complete building.

The stiffness matrices for each tower or frame element were first established and the corresponding lateral stiffness matrices next extracted. The matrices formed by the addition of the appropriate element lateral stiffness matrices were then inverted to give the required lateral flexibility matrices for the whole building.

(e) University Science Buildings.

These relatively rigid buildings are founded on comparatively flexible soils. Hence it proved necessary to include some specific provision for foundation movement in the dynamic analyses.

Providing that equivalent dynamic elastic properties of the soils may be defined, no particular difficulty is encountered in allowing for translational and rotational flexibility(7) and this was done for each of the Zoology, Chemistry and Physics Buildings.

Some model studies were made in order to assess the relative merits of the assumptions made when computing the lateral stiffness of the pierced shear walls and justification for the idealisations eventually chosen was gained in this manner.

The comparatively straightforward framed configuration of the Fray Flats, the Reserve Bank and Wellington Girls' College required no special analysis considerations. Where appropriate, allowance for shear and axial deformation effects and joint size were included.

EXPERIMENTAL TECHNIQUES

The dynamic characteristics of the University buildings were established in a series of small amplitude forced vibration tests. Steady state exciting forces were applied using a specially developed shaking machine(8) and locally designed meters were used to measure the resulting displacements.

By fixing the exciter at roof level and siting horizontal and vertical displacement meters on selected floors the translational and rotational movements of the structure resulting from the applied disturbing forces were determined.

The experimental periods listed in Table 1 for other than the University buildings were determined by a hand-shaking test procedure. This consisted of the generation of an exciting load at the roof level by one or more people swaying from side to side or pushing on a parapet in a similar regular manner, while the extremely small induced lateral displacements of the building were detected and recorded by conventional electronic apparatus. It proved a relatively simple matter to synchronise the exciting motion with the movement of the recording transducer and to thus obtain a reliable indication of the resonant frequency.

RESULTS

The predicted and measured first mode periods of the buildings examined are listed in Table 1.

In the case of the reinforced concrete structures the periods corresponding to fairly high values of elastic modulus are listed so hat a more meaningful comparison between the predicted values and hose obtained from small amplitude shaking tests may be made. onsideration of the shape of concrete stress-strain curves supports he use of a relatively high elastic modulus when making comparisons etween the computed periods and those measured in the manner outlined. Nevertheless it is appreciated that the behaviour under strong motion earthquakes may be better predicted using a lower modulus. An assessment of the effect on the period of a particular choice of elastic modulus may readily be made if it is borne in mind that even halving the elastic modulus will only increase the periods by some 40%.

The period values listed for the Auckland Customs House correspond to the condition in which the piles in the foundation are considered to be 36 feet long and fully fixed at both their upper and lower ends. Periods of 0.81 seconds and 0.63 seconds for the north-south and east-west directions respectively were determined when the tower was considered to be on a fixed base.

The Jellicoe Towers north-south period corresponds to a purely translational movement. The east-west period listed is the first mode value for the coupled translational and rotational movement. It is predominantly translational in nature whereas the second mode in this case proved to be almost entirely rotational with a period of 0.24 seconds. The period of pure torsional oscillation of the unstepped tower was calculated to be 1.33 seconds.

The periods attributed to the Reserve Bank were derived using stiffnesses computed for uncased sections at the preliminary design stage. The final design of the building in fact differs in many significant aspects from the preliminary proposal and this will have the effect of reducing the periods.

The contribution of foundation flexibility to the periods of the University Science buildings is emphasised by the fact that more than one third of the top story displacement in the first mode of vibration is contributed by foundation movement.

COMMENTS ON RESULTS

Good correlation is evident between the predicted and measured periods listed in Table 1. The Code seismic design coefficients which are defined by the period and the zone in which the building is to be located are also presented in Table 1. Since the slopes of the code design spectra(1) are not steep, small changes in the period will not, in themselves, significantly alter the seismic design coefficient. Nevertheless the seismic design process is complicated by the difficulty in selecting member sizes before the lateral loading is defined when the seismic loading is itself dependent on the building stiffness properties.

Various suggestions of semi-empirical expressions useful in estimating the first mode periods of proposed buildings have been reviewed by Housner & Brady(9). They concluded that the calculated periods of steel framed buildings could be found using the equation

$$T = 1.08 \sqrt{N} - 0.86 \text{ seconds}$$

where T is the period
and N is the number of stories.

Taking N as 14, the effective number of stories in the Reserve Bank framework, this expression gives a period value of 3.2 seconds which is in reasonable agreement with the values listed in Table 1 for the preliminary design proposal.

The expression $T = C N \sqrt{B}$

in which B is the breadth of the building, normal to the direction of vibration, proposed by Housner & Brady for computing the period of shear wall type buildings was transposed and used to determine values of the period coefficient C for the University Science Buildings corresponding to the values of period listed in Table 1. The variation, in period coefficient presented in Table 2 supports Housner & Brady's conclusion that "none of the (foregoing) simple equations give satisfactory estimates of the periods and . . . . that good estimates can only be obtained if the actual wall stiffnesses are taken into account."

Their recommendation that the period of a proposed building should be computed by Rayleigh's method possibly prompted the inclusion of the Rayleigh formula in a recent revision clause in the New Zealand Building Code(10).

A similar investigation of the range of period coefficients obtained by inverting the expressions

$$T = C N$$

and

$$T = C \sqrt{N}$$

which have been proposed as suitable for use for space-frame buildings, was undertaken for the three framed structures examined. Once again the large range of period coefficients listed in Table 2 supports the generally held belief that, for this type of building, no single equation can be derived which will give results having possible errors within 100% of the true value.

Not all the buildings described have been built. Some are under construction and others have yet to be started. It is hoped to obtain measurements of the dynamic properties of all of them in due course. Not only will this information be of use in critical examinations of the analysis techniques used in the design process but the measured frequencies provide a good indication of the effective stiffness of buildings. This knowledge could be particularly useful in that further tests following an earthquake would yield results from which an indication of the degree of damage sustained could be estimated.

CONCLUSIONS

It is evident that the currently fashionable style of New Zealand high-rise building, incorporating one or more shear wall, tower or pierced wall elements, is considerably stiffer than estimates based on the results of investigations carried out on structures similar over-all dimensions, in other countries, would lead a designer to expect. To some extent this is probably due to the designers' conscious attempts to provide sufficient rigidity to avoid excessive secondary damage in minor earthquakes but it is possibly also an unintended results of the form of construction favoured locally. The comparatively higher periods predicted for the two pure-frame reinforced concrete structures supports this view.

The importance of making suitable allowance for foundation flexibility is amply demonstrated by the dependence, in several cases, of the predicted dynamic behaviour on the subsoil properties postulated. Experimental evidence of rotational and translational foundation flexibility enables the soil property assumptions at the design stage to be substantiated.

Reasonable correlation exists between the calculated and measured elastic dynamic properties of the typical New Zealand buildings examined, and thus designers appear justified in placing confidence in the analysis methods used at the prediction stage.

ACKNOWLEDGEMENTS

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REFERENCES

1.		1965	N.Z.S.S. 1900, Chapter 8: Basic Design Loads. Standards Association of New Zealand, Wellington, N.Z.
2.	Skinner R.I.	1964	Earthquake Generated Forces and Movements in Tall Buildings. Bulletin 166, D.S.I.R., Wellington, N.Z.
3.	Shepherd R.	1967	Lateral Load Analysis of the Auckland Customs House. New Zealand Engineering 22(7), 273-277.
4.	Shepherd R. & Donald R.A.H.	1967	Seismic Response of Torsionally Unbalanced Buildings. Journal of Sound & Vibration 6(1), 20-37.
5.	Shepherd R.	1967	Predictions of the Response of a Torsionally Unbalanced High-Rise Building to Earthquake Loading. Proceedings, First Australasian Conference on Mechanics of Structures & Materials, University of New South Wales, Sydney, Australia. 16-31.
6.	Shepherd R.	1966	The Dynamic Analysis of an Apartment Building. Bulletin of the Seismological Society of America. 56(1), 13-34.
7.	Shepherd R. & Donald R.A.H.	1967	Foundation Deformation Effects in Structural Dynamic Analysis. Proceedings, Fifth Australia-New Zealand Conference on Soil Mechanics & Foundation Engineering. Auckland, New Zealand.
8.	Shepherd R. & Reay A.M.	1967	Some Apparatus for the Small Amplitude Dynamic Testing of Multistorey Buildings. "Strain". 3(4), 16-21.
9.	Housner G.W. & Brady A.G	1963	Natural Periods of Vibration of Buildings. Proceedings A.S.C.E., 89(EM4), 31-65.
10.	-	1967	Amendment No. 1. Commentary on Chapter 8 of N.Z.S.S. 1900 (MP 12:1965). Standards Association of New Zealand, Wellington, N.Z.

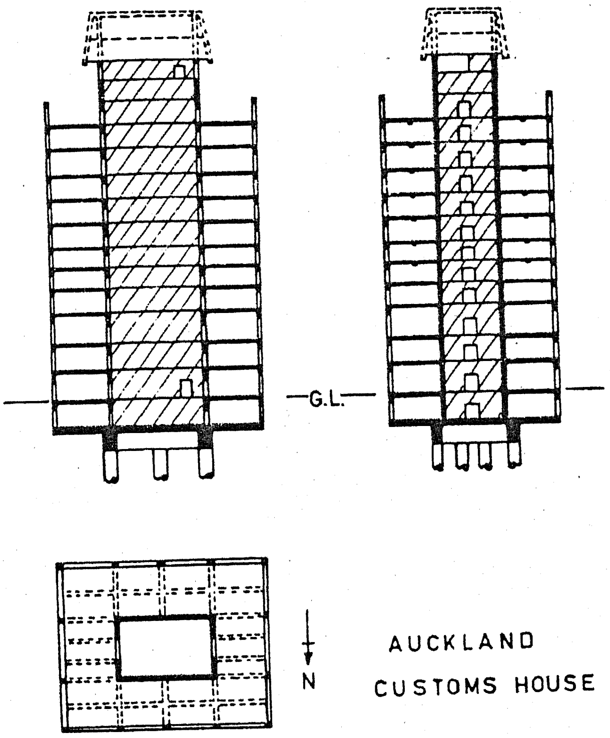
TABLE 1. PROPERTIES OF TYPICAL NEW ZEALAND MULTISTORY BUILDINGS

(Note: Figures in parentheses refer to experimentally determined periods.)

BUILDING	TYPE OF STRUCTURE	HEIGHT	NUMBER OF STORIES	YOUNG'S MODULUS $\times 10^6$ p.s.i.	PERIOD SECONDS MODE 1		ZONE	SEISMIC DESIGN COEFFICIENT	
					N/S	E/W		N/S	E/W
Auckland Customs House	R.C. Tower	178'-0"	15	5.0	1.28	1.13	C	0.04	0.04
Fray Flats	R.C. Frames	127'-0"	13	5.0	0.88	0.86	C	0.06	0.06
Jellicoe Towers Wellington	R.C. Tower	157'-6"	18	5.0	1.18	0.86	A	0.08	0.115
Jerningham Apartments, Wellington	R.C. Spandrel-Beam Frames	120'-9"	14	6.0	0.54 (0.50)	0.49 (0.50)	A	0.15	0.155
Reserve Bank Wellington (Initial Proposal)	Steel Frames	209'-0"	16	30	2.78	2.84	A	0.08	0.08
Terrace Chambers Wellington	Towers + Spandrel Frames	140'-0"	14	5.0	0.70	0.51	A	0.13	0.15
University Science Buildings, Christchurch Zoology	R.C. Shear Walls and Pierced Walls	85'-6"	6	5.2	0.24 (0.24)	0.33 (0.33)	B	0.12	0.12
Chemistry		108'-6"	8	5.2	0.31 (0.31)	0.39 (0.37)	B	0.12	0.12
Physics		108'-6"	8	5.2	0.31 (0.32)	0.38 (0.38)	B	0.12	0.12
Wellington Girls' College	R.C. Frames	76'-5"	6	5.0	0.53		A	0.15	

TABLE 2 VALUES OF PERIOD COEFFICIENT C

BUILDING	TYPE	EXPRESSION	C	
Zoology	Shear Wall	$C = \frac{T}{N\sqrt{B}}$	0.004	0.006
Chemistry	Shear Wall		0.003	0.006
Physics	Shear Wall		0.004	0.006
Fray Flats	Frame	$C = \frac{T}{N}$	0.068	0.066
Jerningham Apartments	Frame		0.039	0.035
Wellington Girls' College	Frame		0.088	
Fray Flats	Frame	$C = \frac{T}{\sqrt{N}}$	0.24	0.24
Jerningham Apartments	Frame		0.14	0.13
Wellington Girls' College	Frame		0.22	



AUCKLAND CUSTOMS HOUSE

FIG. 1

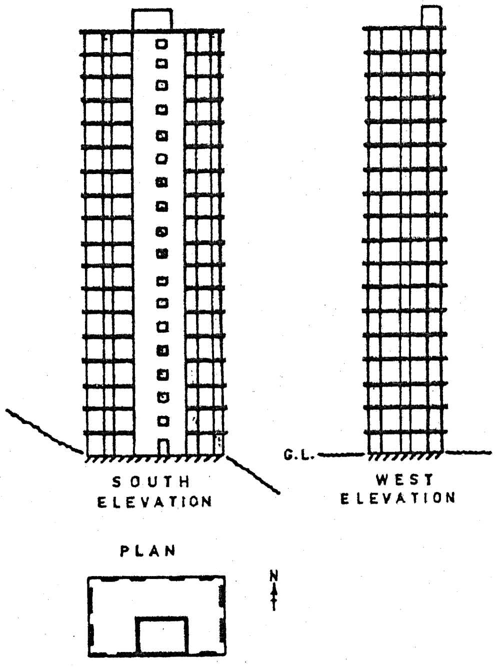


FIG. 2 JELLICOE TOWERS

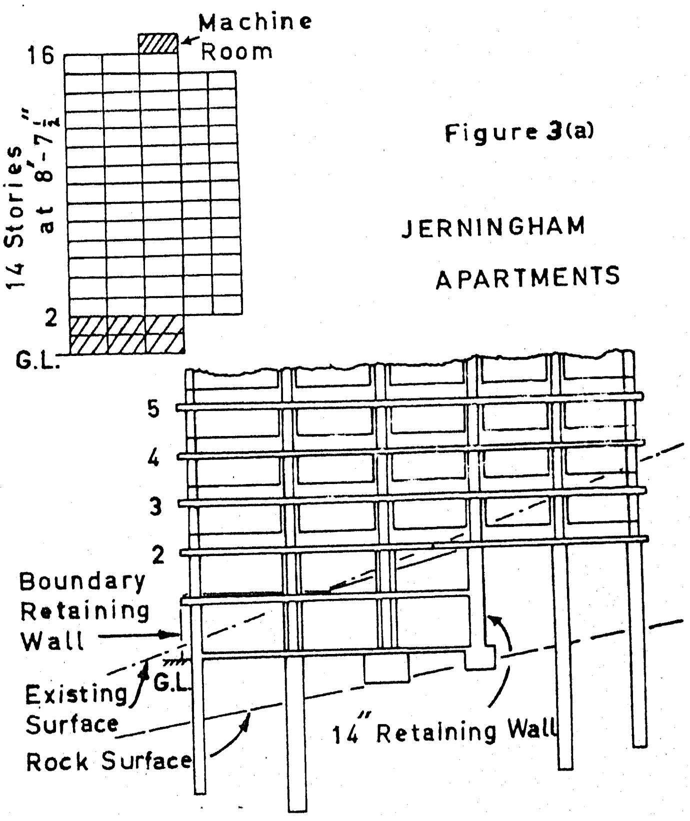


Figure 3(a)

JERNINGHAM APARTMENTS

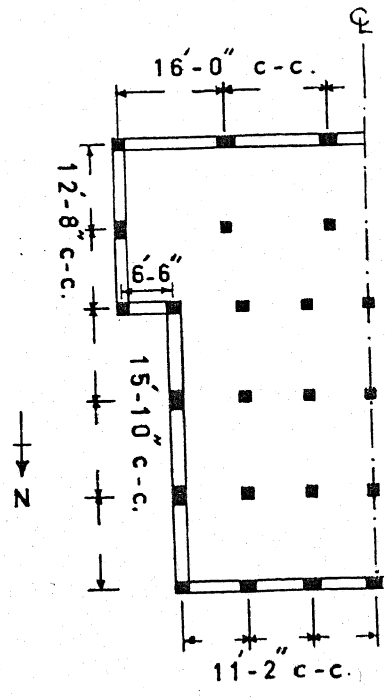


Figure 3(b)

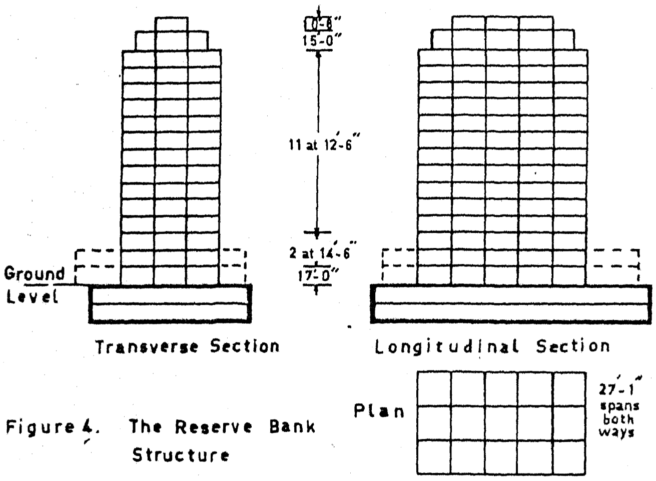


Figure 4. The Reserve Bank Structure

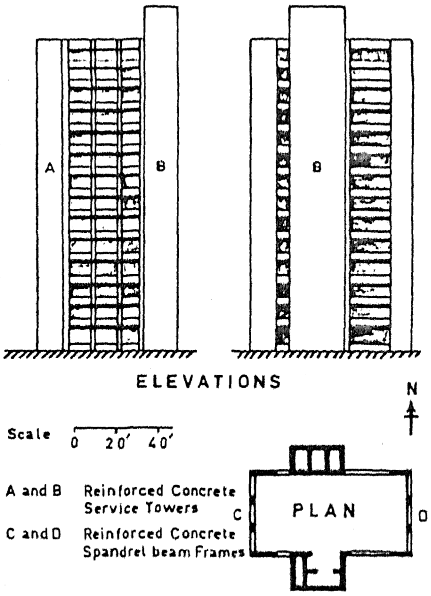
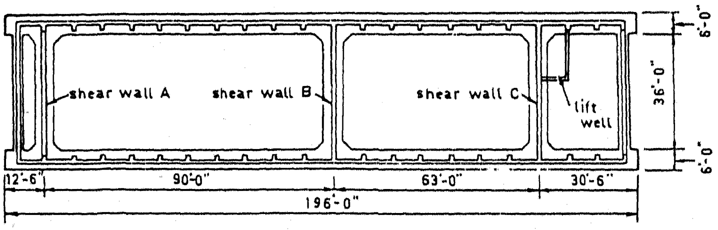
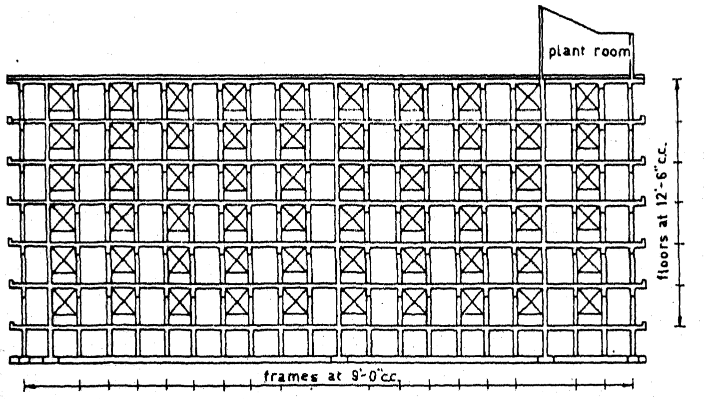
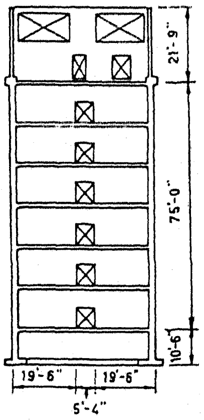


Fig. 5 TERRACE CHAMBERS



FOUNDATION PLAN.





SHEAR WALL C

Fig. 6. Zoology Building.